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REPORT ON GEOTECHNICAL INVESTIGATION

for

PROPOSED ALTERATIONS AND ADDITIONS

at

10 MANOOKA PLACE, WARRIEWOOD, NSW

Prepared For

Timothy & Danielle O'Callaghan

Project No.: 2021-144

July, 2021

Document Revision Record

Issue No	Date	Details of Revisions
0	12th July 2021	Original issue

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1(a) - Checklist of Requirements for Geotechnical Risk Management Report for Development Application

Development Application for ____

Name of Applicant

Address of site __10 Manooka Place, Warriewood, NSW ____________

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: Geotechnical Site Investigation for Proposed Alterations and Additions
Report Date: 12th July 2021 **Project No.:** 2021-144
Author: J. Dee and K.Nicholson
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

■ Comprehensive site mapping conducted ___30th June 2021________ (date)
■ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
■ Subsurface investigation required
- □ No Justification ……………………………………………………..
- ■ Yes Date conducted 30th June 2021…………

■ Geotechnical model developed and reported as an inferred subsurface type-section
■ Geotechnical hazards identified
- □ Above the site
- ■ On the site
- □ Below the site
- □ Beside the site

■ Geotechnical hazards described and reported
■ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Consequence analysis
- ■ Frequency analysis

■ Risk calculation
■ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
■ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
■ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
■ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
■ Design Life Adopted:
- ■ 100 years
- □ Other …………………………………………… specify

■ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
■ Additional action to remove risk where reasonable and practical have been identified and included in the report.
□ Risk assessment within Bushfire Asset Protection Zone.

I am aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ……………………………………………

Name …**Troy Crozier**……………………………

Chartered Professional Status…**RPGeo (AIG)**………

Membership No. …**10197**…

Company… **Crozier Geotechnical Consultants**

REGISTERED PROFESSIONAL GEOSCIENTIST
AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
10197

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for ______________________	
Name of Applicant	
Address of site	10 Manooka Place, Warriewood, NSW

Declaration made by geotechnical engineer or engineering geologist or coastal engineer (where applicable) as part of a geotechnical report

*I, __**Troy Crozier**__ on behalf of ___**Crozier Geotechnical Consultants*** _on this the 12th June 2021_ certify that I am a ~~geotechnical engineer or~~ engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above ~~organisation/~~company to issue this document and to certify that the ~~organisation/~~company has a current professional indemnity policy of at least $2million. I:

☐ have prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

■ am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

☐ have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.

☐ have examined the site and the proposed development/alteration in detail and I am of the opinion that the Development Application only involves Minor Development/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.

☐ have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.

☐ have provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Geotechnical Site Investigation for Proposed Alterations and Additions

Report Date: 12th July 2021 **Project No.:** 2021-144

Author: J. Dee and K. Nicholson

Author's Company/Organisation: Crozier Geotechnical Consultants

Documentation which relate to or are relied upon in report preparation:

Architectural Drawings by Newbuild Design and Drafting, DWG No.: 370-2, Dated: 04/21
Survey Drawing by Waterview Surveying Services, Project No.: 1357, Revision A, Dated: 07/06/2021

I am aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ensuring that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ....

Name ...**Troy Crozier**..........................

Chartered Professional Status...**RPGeo (AIG)**........

Membership No. ...**10197**..........................

Company... **Crozier Geotechnical Consultants**

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Date: 12th July 2021
Project No: 2021-144

GEOTECHNICAL REPORT FOR PROPOSED ALTERATIONS & ADDITIONS 10 MANOOKA PLACE, WARRIEWOOD, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for proposed alterations and additions at 10 Manooka Place, Warriewood, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request Newbuild Design and Drafting on behalf of the clients Ken and Danielle O'Callaghan.

It is understood that alterations and additions to the site house are proposed including an extension to the rear of the house which will involve excavation into an existing slope.

The site is located within the H1 (highest category) landslip hazard zone as identified within Northern Beaches Councils precinct (Geotechnical Risk Management Policy for Pittwater - 2009). For Development Application purposes, to meet the Councils Policy requirements for land classified as H1 a detailed Geotechnical Report which meets the requirements of Paragraph 6.5 of that policy must be submitted. This report must include a landslide risk assessment to the methods of AGS 2007 for the site and proposed works, plans, geological sections and provide recommendations for construction and to ensure stability is maintained for a preferred design life of 100 years.

This report includes a description of site and subsurface conditions, borehole logs, in-situ test results, a geotechnical assessment of the ground conditions underlying the site, site mapping/plan, a geological cross section, a landslide risk assessment of the site and proposed works and recommendations for preliminary design and construction.

The investigation and reporting were undertaken as per the Proposal No.: P21-298, Dated: 16th June 2021.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.

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b) Drilling of two boreholes using hand tools along with six Dynamic Cone Penetrometer (DCP) tests to investigate the subsurface geology, depth to bedrock and identification of ground water conditions.

The following plans and drawings were supplied for the work:

- Architectural Drawings – Newbuild Design and Drafting, DWG No.: 370-2, Dated: 04/21
- Survey Drawing – Waterview Surveying Services, Project No.: 1357, Revision A, Dated: 07/06/2021

1.1 Proposed Development

It is understood that the proposed works involve additions and alterations to the existing residence that will include a northern extension of the site house. The proposed extension will be two storeys in height and will extend the house footprint by approximately 2.5m to the north and into the rear garden. The rear of the existing house is approximately 1.0m to the south of a rock cutting which will require excavation to accommodate the extension. The maximum height/depth of the excavation appears to be approximately 3.50m and will extend to within 1.0m of the western boundary and >10.0m from all other shared boundaries.

2. SITE FEATURES:

2.1. Description:

The site is a broadly triangular shaped block located on the high northern side Manooka Place at the end of a cul-de-sac. The site has rear north, side west/east and front southern boundaries of 37.10m, 35.76m, 38.23m and 13.46m respectively and covers an area of approximately 702m^2 as referenced from the provided survey plan. Site surface levels reduce from north to south from a high of RL49.50m to a low of RL37.50m.

An aerial photograph of the site and its surrounds is provided below (Photograph 1), as sourced from NSW Government Six Map spatial data system which includes the boundary designations assigned and used throughout this report for clarity.

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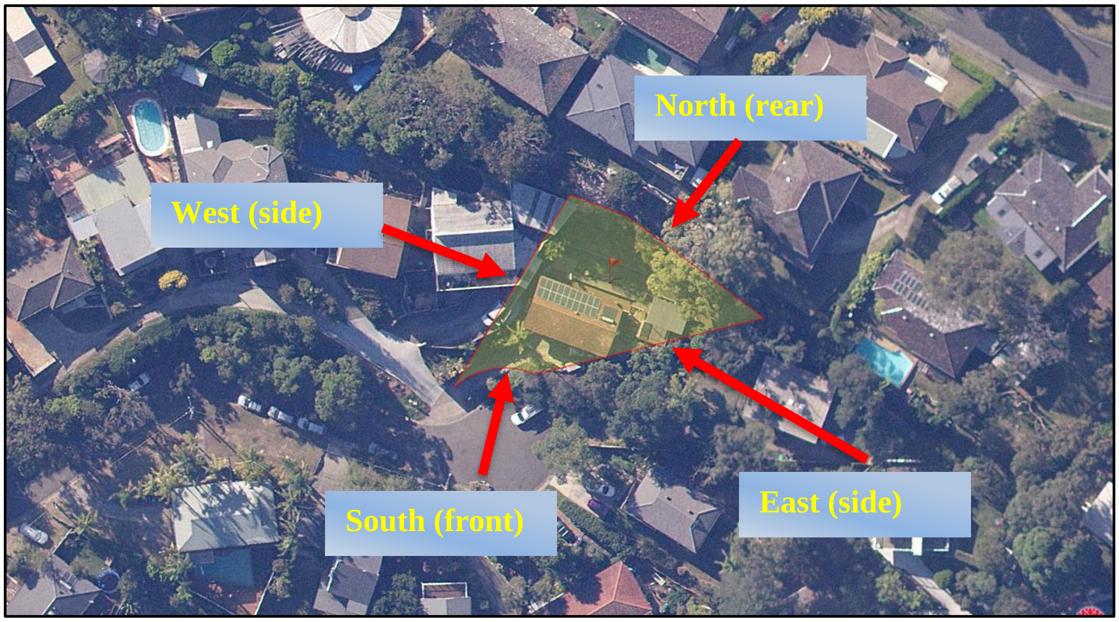

Photograph 1: Aerial photo of site and surrounds

The site dwelling comprises a two storey brick (lower level) and timber clad (upper level) residence with a steep concrete driveway and staircase linking it to the road with timber two decks (front and rear). The rear garden is stepped with a series of timber and stone retaining walls supporting two terraced lawns. General views of the site are provided in Photographs 2 & 3.

Photograph 2: View of the site residence looking broadly north-east from the driveway

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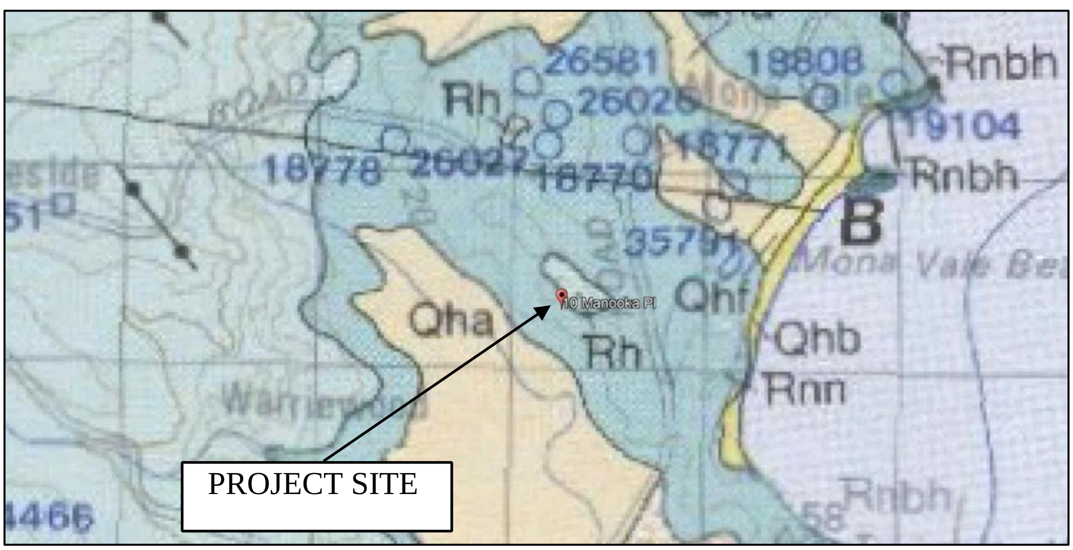
Photograph 3: View of the rear of the property looking south-west

2.2. Geology:

Reference to the Sydney 1:100,000 Geological Series sheet 9130) indicates that the site is underlain by Newport Formation (Upper Narrabeen Group) rock (Rnn) which is of middle Triassic Age. The Newport Formation typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones that tend to deep weathering and produce a weak rock mass containing clay bands and fracturing. Deposits of the Hawkesbury Sandstone are shown to the east (upslope) of the site. An extract from the relevant Geological Series sheet is provided below.



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3. FIELD WORK:

3.1. Methods:

The field work comprised a ‘walkover’ inspection/mapping of the site and a subsurface investigation, both of which were undertaken/supervised by a Geotechnical Engineer on 30 June 2021.

The walkover comprised inspection of existing structures, slopes, rock outcrops and vegetation within and adjacent to the site to assist with identification of potential geotechnical hazards which may impact the development.

The subsurface investigation comprised the drilling of two boreholes (BH1 and BH2). The boreholes were undertaken using hand auger techniques due to access restrictions.

DCP testing was carried out from the ground surface adjacent to the boreholes in accordance with AS1289.6.3.2 – 1997, “Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer”.

The borehole locations were determined on site by CGC by direct measurement from existing site features and are indicated approximately on the attached Figure 1, (Appendix 1). The ground surface level at each borehole location was interpolated by reference to the provided survey plan.

Prior to commencement of drilling, underground service location was undertaken by an accredited service locator. A Sydney Water asset within the rear of the site was not detectable using radio detection techniques.

Disturbed soil samples were recovered from the auger for geotechnical logging purposes which was undertaken in accordance with AS1726:2017 ‘Geotechnical Site Investigations’.

On completion the boreholes were backfilled with arisings and surface compacted.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed Borehole Log sheets and Dynamic Penetrometer Test Sheet in Appendix: 2. A geological model/section is provided as Figure: 2, Appendix: 2.

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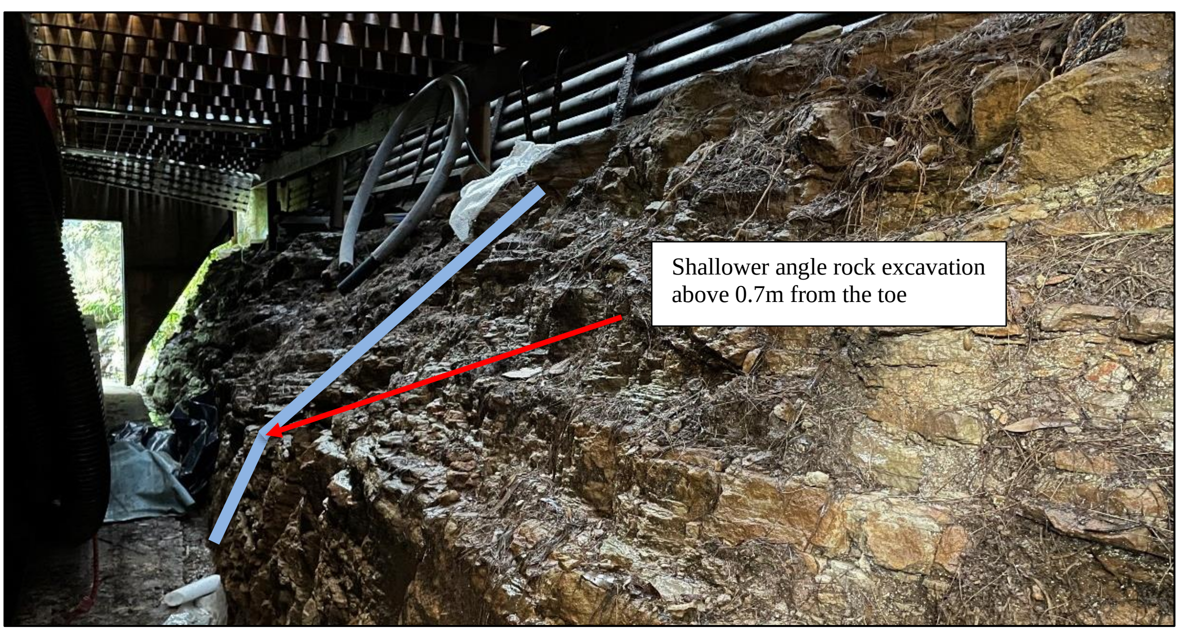
3.2. Field Observations:

The site is situated on the high north side of Manooka Place within moderately then steeply south dipping topography. The site residence is accessed via a concrete driveway which adjoins Manooka Place to the south-west of the site and provides access to a carport underneath the dwelling. Minor settlement of Manooka Place roadway was visible in the south (downslope) external to the site however the concrete driveway and stairs showed no signs excessive settlement or cracking.

The site residence comprises a two-storey brick and timber clad residence with carport under which appears to be of approximately 1990's style construction and shows no signs of excessive settlement or cracking visible on the external walls.

A concrete pathway wraps around the structure at ground level which provides access to the rear of the property and bedrock cutting. The cutting (shown in Photograph 4) varies in height between approximately 1.4m (south) and 2.0m (north) and extends along the full width of the rear of the site residence. The strata comprised sandstone bedrock with minor shale/siltstone lenses and appears to range from very low to medium strength. Near the base of the cutting (within the lower 0.7m), the bedrock appears stronger (low to medium strength typically) and massively bedded. Within the upper 1.5m (approximately) the frequency of both bedding and jointing defects increases, and the cutting has been excavated at a shallower batter slope angle than the lower, near vertical basal section. Some localized minor wedge type detached cobbles of sandstone were visible within the upper face however the cutting appeared globally stable with no significant weak or unstable zones observed. Groundwater was observed emanating from the rock face through defects.



Photograph 4: View of the exposed rock rack abutting the northern edge of the dwelling

A timber retaining wall is founded near the crest of the cutting. Retaining walls surround the house structure to the east and west which appeared in good condition.

Photograph 5 and 6: Western and eastern sides of the existing dwelling looking north

The neighbouring property upslope to the north (No. 37 Bertana Crescent) contains a two storey brick residential apartment block approximately 20m from the shared boundary. The ground surface along the shared boundary is similar to the site with the remainder of the block increasing in eleveation to the north. The dwelling did not display any obvios signs of cracking or excessive settlement and no credible landslip hazards were observed on the rest of the property.

The neighbouring property to the east (No. 11 Manooka Place) contains a two storey brick and timber residence approximately 15.0m from the shared boundary with a large vegetated garden making up the majority of the property. The property shares similar ground levels as the site along the shared boundary and follows a similar south dipping topography. Observations were limited due to the dense vegetation however no significant indications of impending geotechnical hazards where observed within No.11.

The neighbouring property to the west of the site (No. 9 Manooka Place) contains a one and two storey rendered residence situated approximately 4.0m from the shared boundary. The property also contains two smaller structures constructed immediately adjacent to the boundary; a small shed situated alongside the proposed site excavation and a covered terrace area towards the rear of the property. Whilst the property shares similar ground levels across rear northern portion of the shared boundary adjacent to the sites rear garden, the shared boundary alongside the site house is made up of a large block retaining wall of varying height (2.0m-3.0m) with the neighbouring property sitting at significantly higher RL's. The rear (north) portion of the retaining wall appears to be founded on the same sandstone outcrop formation observed behind the site house. The retaining wall did not show any signs of excessive settlement or rotation. All structures within this property appeared in good condition with no external cracking or indications of geotechnical instability.

The neighbouring properties and structures were inspected from the site or road reserves, however visible aspects showed no indications of geotechnical hazard that may impact the site.

3.3. Ground Conditions:

The boreholes (BH1 and BH2) were drilled using a hand auger with both refusing in very stiff to hard medium plasticity clay at a depth of 0.65m.

DCP tests were carried out from the ground surface adjacent to the boreholes and at an additional four locations with refusal encountered on interpreted bedrock at a maximum depth of 1.4m.

Based on the borehole logs and DCP test results, the subsurface conditions at the site can be classified as follows:

- **TOPSOIL/FILL –** Topsoil/Fill was encountered in both boreholes to a maximum depth of 0.3m. The fill predominately comprised of a dark brown silty sandy topsoil with sandstone and ironstone gravels.
- **CLAY –** Natural clay soils were recovered from both boreholes below a depth of 0.3m. The soil comprised of firm orange-brown medium platicity clay grading to hard with ironstone gravels and is likely to contain zones of extremely weathered bedrock.
- **BEDROCK –S**andstone bedrock of at least very low strength was interpreted at all locations from a minimum depth of 0.10m (DCP5) and a maximum depth of 1.45m (DCP6).

A freestanding groundwater table or significant seepage was not identified at any of the test locations however was observed emanating from the rock outcrop.

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4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified the presence of predominantly clay soils across the site beneath a shallow layer of topsoil/fill. This layer extends to a maximum depth of 1.40m below ground level before sandstone bedrock is intercepted which is expected to increase in strength to low to medium strength however localised weak zones may be present within the upper section of rock cutting.

It is understood that the proposed excavation will be of approximately 3.5m below the existing ground level within the rear garden that will extend to within 1.0m of the shared western boundary. The distance to the other shared boundaries is such that the hazards/risks associated are negligible.

The stability of the excavation and ground vibrations as a result of rock excavation equipment are considered to be the key geotechnical components of the proposed works.

Based on the investigation results and site topography, the excavation is expected to encounter fill soils then stiff to hard natural clay soils to a maximum depth of approximately 1.40m before intersecting bedrock. This bedrock is likely to be of very low strength however it is anticipated to grade quickly to low to medium strength. Safe batter slopes within the site appear feasible on all sides of the excavation without adverse impacts to adjacent neighboring structures except the shared boundary to the west. Within this area, the north-west corner of the proposed excavation is approximately 1.0m from the shared boundary with No. 9 Manooka Place. It is envisaged pre-excavation support will be required to maintain boundary stability where safe temporary batters (as provided in Section 4.3) cannot be implemented. This could be achieved by incrementally excavating and providing support such that no face greater than approximately 1.5m in width is left unsupported during (and following) the excavation process. Alternatively, a bored pile wall (or similar) could be constructed prior to excavation. Where the existing site structures are to remain (e.g., retaining walls in rear garden) the footing depth/conditions should be determined to ensure adverse impacts resulting from the required excavation.

Sandstone bedrock of at least low strength can be excavated at steep to vertical batter slopes provided it is unfractured by the excavation works and does not contain unfavorable defects. Where these are encountered then support systems (i.e. rock bolts/shotcrete) can be implemented as excavation works progress. There were limited stability hazards identified in the investigation. However, increased defects were observed within the upper section of the rock cutting which may result in localized rockslide/topple failure with potential impact to the site. Therefore, geotechnical inspection following initial clearing of the bedrock surface is required to confirm site conditions along with inspection at regular depth intervals during excavation. Through selection

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of suitable excavation equipment, geotechnical inspection and mapping during the excavation works along with the installation of support measures as determined necessary by the inspections, the risk from the proposed works can be maintained within ‘Acceptable’ levels for all situations (AGS 2007).

The excavation of low up to high strength rock requires the use of rock excavation equipment which can produce ground vibrations of a level which can potentially cause damage to neighbouring structures. Therefore, selection of suitable equipment and a sensible methodology are critical. The need for full time vibration monitoring will be determined based upon the type of rock excavation equipment proposed for use. It is recommended that a rock saw and small (≤250kg) rock hammers be proposed for use at this site to avoid the need for full time monitoring. Crozier Geotechnical Consultants should be consulted for assessment of the proposed equipment prior to its use. Where closely spaced defects are encountered in excavation, the need for rock excavation equipment and/or vibration monitoring may be reduced however this would require additional rock core boreholes to confirm or assessed as excavation progresses.

A ground water table is not expected to be encountered within the proposed depth of excavation(s) however seepage at the interface between fill/residual soils and the soil/bedrock contact is anticipated as well as seepages through he exposed bedrock face.

New footings should be founded off bedrock of uniform strength to provide resistance against any lateral movement and to reduce the potential for variation in settlement under individual footings.

The founding conditions of the existing footings are unknown, therefore where significant increase in load is proposed to old footings as a result of the works then the founding conditions and footing style should be further assessed.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or neighbouring properties provided the recommendations of this report are implemented in the design and construction phases.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and hand auger boreholes. This test equipment provides limited data from small, isolated test points across the entire site with limited penetration into rock, therefore some minor variation to the interpreted subsurface conditions is possible, especially between test locations. However, the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent initial design of the proposed works.

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4.2. Site Specific Risk Assessment:

Based on our site investigation we have identified the following geological/geotechnical landslip hazards which need to be considered in relation to the existing site and the proposed works. The hazards are:

A. Earth slide (<5m^3) of fill and residual soils at the crest of the excavation.
B. Rockslide/ topple (<5m^3) of bedrock around the perimeter of the excavation
C. Rockslide/ topple (<1m^3) of bedrock around the perimeter of the excavation

A qualitative assessment of risk to life and property related to these hazards is presented in Table A and B, Appendix: 3, and is based on methods outlined in Appendix: C of the Australian Geomechanics Society (AGS) Guidelines for Landslide Risk Management 2007. AGS terms and their descriptions are provided in Appendix: 4.

Due to the separation distances of the proposed excavation to property boundaries and the elevation of neighbouring properties, it is considered that landslide hazards/ risks because of the proposed development are limited to No. 9 Manooka Place and the onsite structures.

The Risk to Life from Hazard A was estimated to be up to **7.81 x 10^{-6},** whilst the Risk to Property was considered to be **'Low'**. The hazard was therefore considered to be 'Acceptable' when assessed against the criteria of the AGS 2007 and the Geotechnical Risk Management Policy.

The Risk to Life from Hazard B was estimated to be up to **4.17 x 10^{-6},** whilst the Risk to Property was considered to be **'Moderate'**. The hazard was therefore considered to be 'Unacceptable' when assessed against the criteria of the AGS 2007 and the Geotechnical Risk management Policy.

The Risk to Life from Hazard C was estimated to be up to **1.04 x 10^{-6},** whilst the Risk to Property was considered to be **'Low'**. The hazard was therefore considered to be 'Unacceptable' when assessed against the criteria of the AGS 2007 and the Geotechnical Risk management Policy.

The above risk to life and property from Hazard A, Hazard B and Hazard C have been assessed assuming insufficient retention systems are constructed within the site. Where appropriate retention systems are installed, the anticipated risks are expected to further reduce.

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4.3. Design & Construction Recommendations:

Design and construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	- Class 'A' for footings on bedrock at base of excavation or off bedrock surface
Type of Footing	Strip/pad, Slab or Pier
Sub-grade material and Maximum Allowable Bearing Capacity	- Weathered, VLS Sandstone: 800kPa - Weathered LS Sandstone: 1000kPa - Weathered MS Sandstone: 2000kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e – rock site

Remarks:

All footings should be founded off bedrock of similar strength to prevent differential settlement.

All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project.

4.3.2. Excavation:	
Depth of Excavation	up to 3.50m depth

Property Separation:

The table below shows the properties potentially affected by the proposed excavation, excavation depth and the separation distances to the shared property boundary/structure.

Table 1: Property separation distances

Boundary	Property	Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)*	
				Boundary	Structure
North	No. 37 Bertana Crescent	House	3.50	10.00	20.0
East	No. 11 Manooka Place	House		10.00	20.0
South	Manooka Place	Services		14.00	-
West	No. 9 Manooka Place	Shed		1.0	1.5

Type of Material to be Excavated		
	Up to 0.30m	Fill/topsoil
	from 0.30m to 1.40m	Residual clay soils
	from 1.40m	Sandstone bedrock – VLS – MS, potentially HS

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Guidelines for un-surcharged batter slopes for this site are tabulated below:

Material	Safe Batter Slope (H: V)	
	Short Term/Temporary	Long Term/Permanent
Fill and natural soils	1.5:1	2.0:1
Very Low (VLS) strength or fractured bedrock	1:1**	0.5:1*
Medium strength (MS), defect free bedrock	Vertical*	Vertical*

*Dependent on defects and assessment by engineering geologist. ** Based on existing rock cutting

Remarks:

Seepage at the bedrock surface or along defects in the soil/rock can also reduce the stability of batter slopes or rock cuts and invoke the need to implement additional support measures.

Where safe batter slopes are not implemented, the stability of the excavation cannot be guaranteed until permanent support measures are installed. This should also be considered with respect to safe working conditions. Batter slopes should not be left unsupported without geotechnical inspection and approval.

Should further detail on rock strengths or conditions for excavation costing be required, then cored boreholes and laboratory testing will be required.

Equipment for Excavation	Fill/natural soils	Bucket
	VLS bedrock	Bucket and ripper
	LS – MS/HS bedrock	Rock hammer and rock saw

VLS – very low strength, LS – low strength, MS – medium strength, HS – high strength

Remarks:

Rock sawing of the hard rock excavation perimeters is recommended as it has several advantages. It often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal and the saw cuts will provide a slight increase in buffer distance for use of rock hammers. It also reduces deflection across boundary of detached sections of bedrock near surface.

Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength Sandstone bedrock, to achieve a low level of vibration (5mm/s PPV) the below hammer weights and buffer distances are generally required:

Maximum Hammer Weight	Required Buffer Distance from Structure
300kg	2.00m
400kg	3.00m
600kg	6.00m
≥1 tonne	Up to 20.00m

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Onsite calibration and full-time vibration monitoring will provide accurate vibration levels to the site-specific conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Inspection of equipment and review of dilapidation surveys and excavation location is necessary to determine need for full time monitoring. Where monitoring is determined as necessary then it should be maintained directly between the excavation activity and the structure being monitored, as such the monitor may require relocation during excavation.	
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Neighbouring residential dwellings = 5mm/s Services = 3mm/s
Vibration Calibration Tests Required	If larger scale (i.e., rock hammer >250kg) excavation equipment is proposed
Full time vibration Monitoring Required	Pending proposed excavation equipment and vibration calibration testing results, if required
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • Upon initial clearing of soils • Upon installation of excavation support if determined necessary • At 1.50m depth intervals of excavation • At completion of the excavation • Where ground conditions are exposed that differ to those than expected
Dilapidation Surveys Requirement	Recommended on neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage.
Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope.	

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4.3.3. Retaining Structures:	
Required	New retaining structures are proposed to support the excavation perimeters above the rock face.
Types	Steel reinforced concrete/concrete block post excavation (or bored pile walls pre-excavation), designed in accordance with Australian Standards AS4678-2002 Earth Retaining Structures.

Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill and Residual Clay Soils	20	$\phi' = 30°$	0.33	0.47	3.25
ELS to VLS sandstone	22	$\phi' = 38°$	0.15	0.20	200 kPa

Remarks:

In suggesting these parameters, it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided at the rear of the wall footings. If this is not done, then the walls should be designed to support full hydrostatic pressure in addition to pressures due to the soil backfill. It is suggested that the retaining walls should be backfilled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted to minimize horizontal stresses.

Retaining structures near site boundaries or existing structures should be designed with the use of at rest (K_0) earth pressure coefficients to reduce the risk of movement in the excavation support and resulting surface movement in adjoining areas. Backfilled retaining walls within the site, away from site boundaries or existing structures, that may deflect can utilize active earth pressure coefficients (Ka).

4.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation0		Along defects in bedrock
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<0.50L/min), within fill/natural soil interface and at bedrock surface.
Site Location and Topography		High north side of the road, within south dipping topography

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Impact of development on local hydrogeology	Appears negligible
Onsite Stormwater Disposal	Not required
Remarks: As the excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, downpipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Council's stormwater system off site.	

4.4. Conditions Relating to Design and Construction Monitoring:

To comply with Councils conditions and to enable us to complete Forms: 2b and 3 required as part of construction, building and post-construction certificate requirements of the Councils Geotechnical Risk Management Policy 2009, it will be necessary for Crozier Geotechnical Consultants to:

1. Review the structural design drawings for compliance with the recommendations of this report prior to construction,
2. Inspection of site and works as per Section 4.3 of this report
3. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
4. Inspect completed works to ensure construction activity has not created any new hazards and that all retention and stormwater control systems are completed.

The client and builder should make themselves familiar with the Councils Geotechnical Policy and the requirements spelled out in this report for inspections during the construction phase. Crozier Geotechnical Consultants cannot sign Form: 3 of the Policy if it has not been called to site to undertake the required inspections.

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4.5. Design Life of Structure:

We have interpreted the design life requirements specified within Council's Risk Management Policy to refer to structural elements designed to support the existing structures, control stormwater and maintain the risk of instability within acceptable limits. Specific structures and features that may affect the maintenance and stability of the site in relation to the proposed and existing development are considered to comprise:

- stormwater and subsoil drainage systems,
- retaining walls and instability,
- maintenance of trees/vegetation on this and adjacent properties.

Man-made features should be designed and maintained for a design life consistent with surrounding structures (as per AS2870 – 2011 (100 years)). It will be necessary for the structural and geotechnical engineers to incorporate appropriate design and inspection procedures during the construction period. Additionally, the property owner should adopt and implement a maintenance and inspection program.

If this maintenance and inspection schedule are not maintained the design life of the property cannot be attained. A recommended program is given in Table: C in Appendix: 3 and should also include the following guidelines.

- The conditions on the block don't change from those present at the time this report was prepared, except for the changes due to this development.
- There is no change to the property due to an extraordinary event external to this site
- The property is maintained in good order and in accordance with the guidelines set out in;
  a) CSIRO sheet BTF 18
  b) Australian Geomechanics "Landslide Risk Management" Volume 42, March 2007.
  c) AS 2870 – 2011, Australian Standard for Residential Slabs and Footings

Where changes to site conditions are identified during the maintenance and inspection program, reference should be made to relevant professionals (e.g. structural engineer, geotechnical engineer or Council). Where the property owner has any lack of understanding or concerns about the implementation of any component of the maintenance and inspection program the relevant engineer should be contacted for advice or to complete the component. It is assumed that Council will control development on neighbouring properties, carry out regular inspections and maintenance of the road verge, stormwater systems and large trees on public land adjacent to the site to ensure that stability conditions do not deteriorate with potential increase in risk level to the site.

Also, individual Government Departments will maintain public utilities in the form of power lines, water and sewer mains to ensure they don't leak and increase either the local groundwater level or landslide potential.

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5. CONCLUSION:

The site investigation identified the presence residual clay soils overlain by a thin layer of topsoil/fill. The residual clay soils likely grade to weathered bedrock. Based on DCP test results, the depth to the bedrock of minimum very low strength was interpreted to vary from 0.10m to greater than 1.40m over the proposed development site with the rock expected to comprise of sandstone with potential for some interbedded shale. It is expected that the proposed excavation will extend through topsoil/fill, residual soils and very low strength sandstone bedrock which is likely to grade quickly to medium strength (or stronger) sandstone and hard rock excavation equipment is likely to be required.

It is recommended that a preliminary vibration limit (Maximum Peak Particle Velocity, PPV) of 5mm/s PPV be set at the founding level for neighbouring structures for all excavation work on this site to maintain comfort levels and provide a very low probability of structural damage.

The bedrock has the potential for weathered seams and defects which could impact the stability of the excavation. Therefore, geotechnical inspection following initial clearing of the bedrock surface is required to confirm site conditions along with inspections during excavation.

The risks associated with the proposed development can be maintained within 'Acceptable' levels with negligible impact to neighbouring or site structures provided the recommendations of this report, and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

Prepared by:	Reviewed by:
James Dee	Kieron Nicholson
Geotechnical Engineer	Senior Engineering Geologist

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6. REFERENCES:

1. Australian Geomechanics Society 2007, "Landslide Risk Assessment and Management", Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Geological Society Engineering Group Working Party 1972, "The preparation of maps and plans in terms of engineering geology" Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
3. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin
4. Australian Standard AS 3798 – 2007, Guidelines on Earthworks for Commercial and Residential Developments.
5. Australian Standard AS 2870 – 2011, Residential Slabs and Footings – Construction
6. Australian Standard AS1170.4 – 2007, Part 4: Earthquake actions in Australia
7. Australian Standard AS 1726 – 2017, Geotechnical Site Investigations

Appendix 1

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NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigation Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. Sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows:

Classification	Undrained Shear Strength kPa
Very soft	Less than 12
Soft	12 - 25
Firm	25 – 50
Stiff	50 – 100
Very stiff	100 - 200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300mm)	CPT Cone Value (Qc – MPa)
Very loose	less than 5	less than 2
Loose	5 – 10	2 – 5
Medium dense	10 – 30	5 -15
Dense	30 – 50	15 – 25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

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Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling to allow information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Drilling Methods

The following is a brief summary of drilling methods currently adopted by the company and some comments on their use and application.

Test Pits – these are excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descent into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) – the hole is advanced by a rotating plate or short spiral auger, generally 300mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling – the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers – the hole is advanced using 90 – 115mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPT's or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling - the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling – similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. From SPT).

Continuous Core Drilling – a continuous core sample is obtained using a diamond-tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedures is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150mm of say 4, 6 and 7
  as 4, 6, 7 then N = 13
- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm then as 15, 30/40mm.

The results of the test can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50mm diameter thin wall sample tubes in clay. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch Cone – abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australia Standard 1289, Test 6.4.1.

In tests, a 35mm diameter rod with a cone-tipped end is pushed continually into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separte 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected buy electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) their information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: -

- Cone resistance – the actual end bearing force divided by the cross-sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio - the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0 – 5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0 – 50 MPa) is less sensitive and is shown as a full line. The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios 1% - 2% are commonly encountered in sands and very soft clays rising to 4% - 10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range: -

$$Q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N blows (blows per 300mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range: -

$$Q_c = (12 \text{ to } 18)\ C_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculations of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Dynamic Penetrometers

Dynamic penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer – a 16mm diameter flattened rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test 6.3.3). The test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). The test was developed initially for pavement sub-grade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is generally carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

Borehole Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

Details of the type and method of sampling are given in the report and the following sample codes are on the borehole logs where applicable:

D	Disturbed Sample	E	Environmental sample	DT	Diatube
B	Bulk Sample	PP	Pocket Penetrometer Test		
U50	50mm Undisturbed Tube Sample	SPT	Standard Penetration Test		
U63	63mm " " " " "	C	Core		

Ground Water

Where ground water levels are measured in boreholes there are several potential problems:

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made. More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. A three-storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty-storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions – the potential for this will depend partly on bore spacing and sampling frequency,
- changes in policy or interpretation of policy by statutory authorities,
- the actions of contractors responding to commercial pressures,

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

Site Anomalies

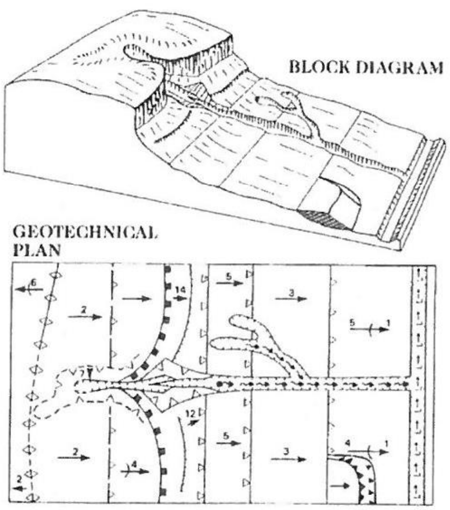
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

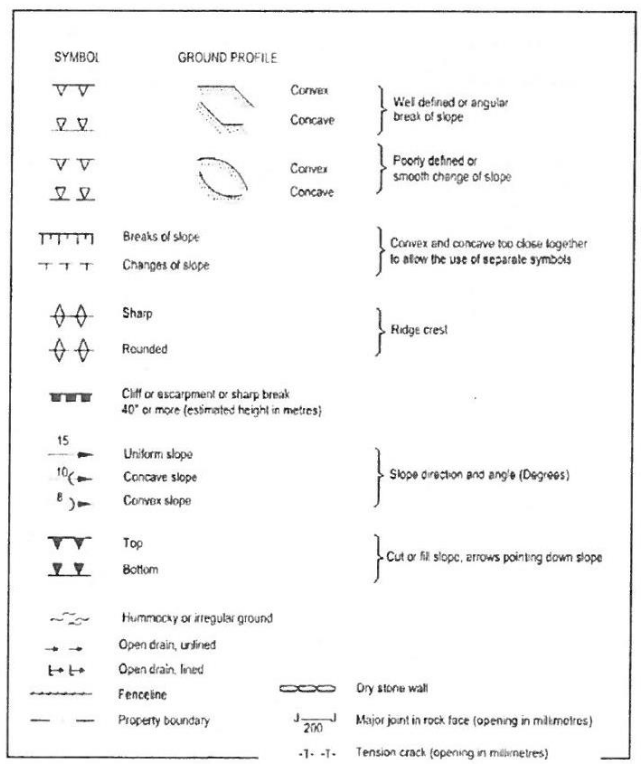
Reproduction of Information for Contractual Purposes

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a special ally edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

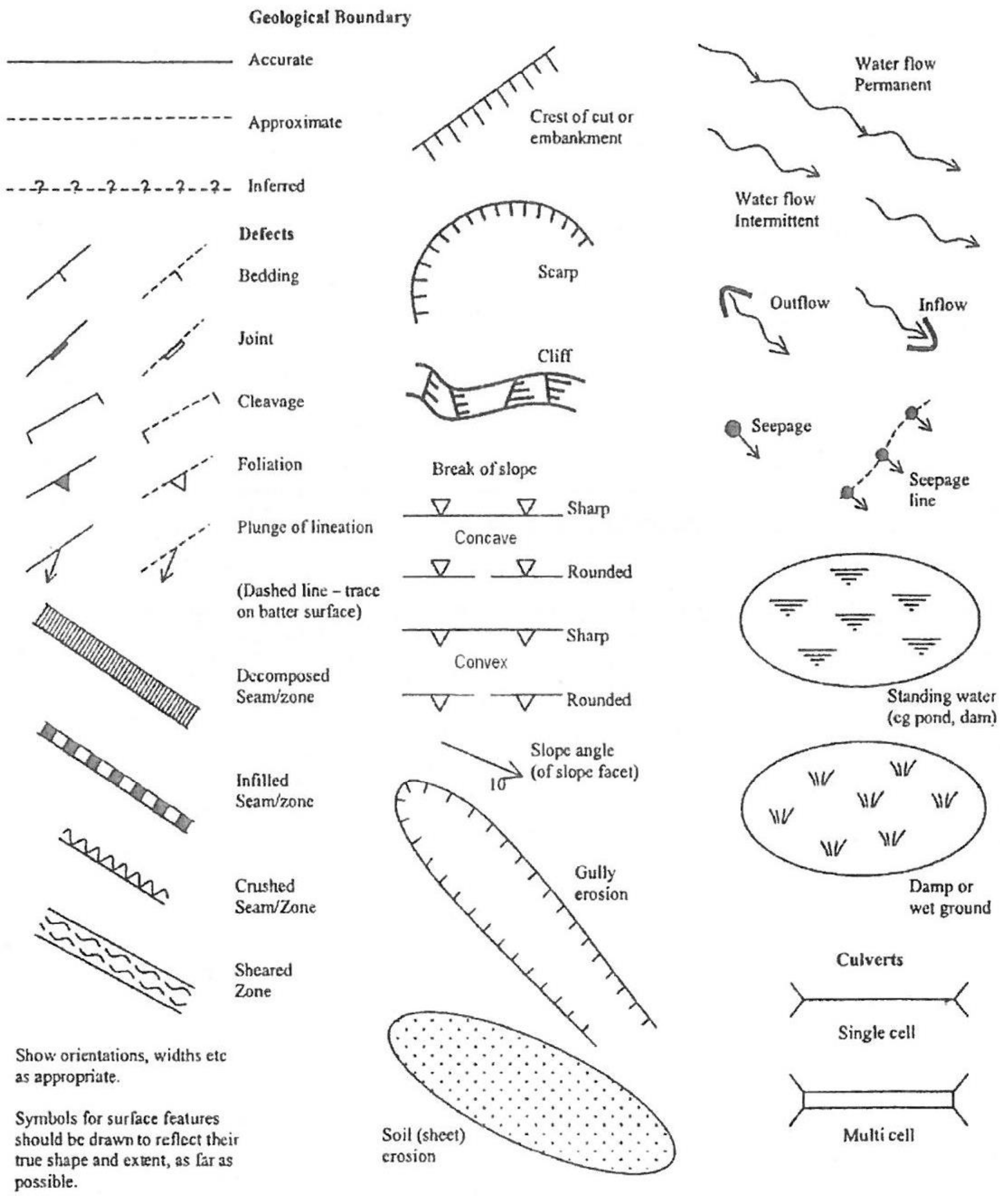
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





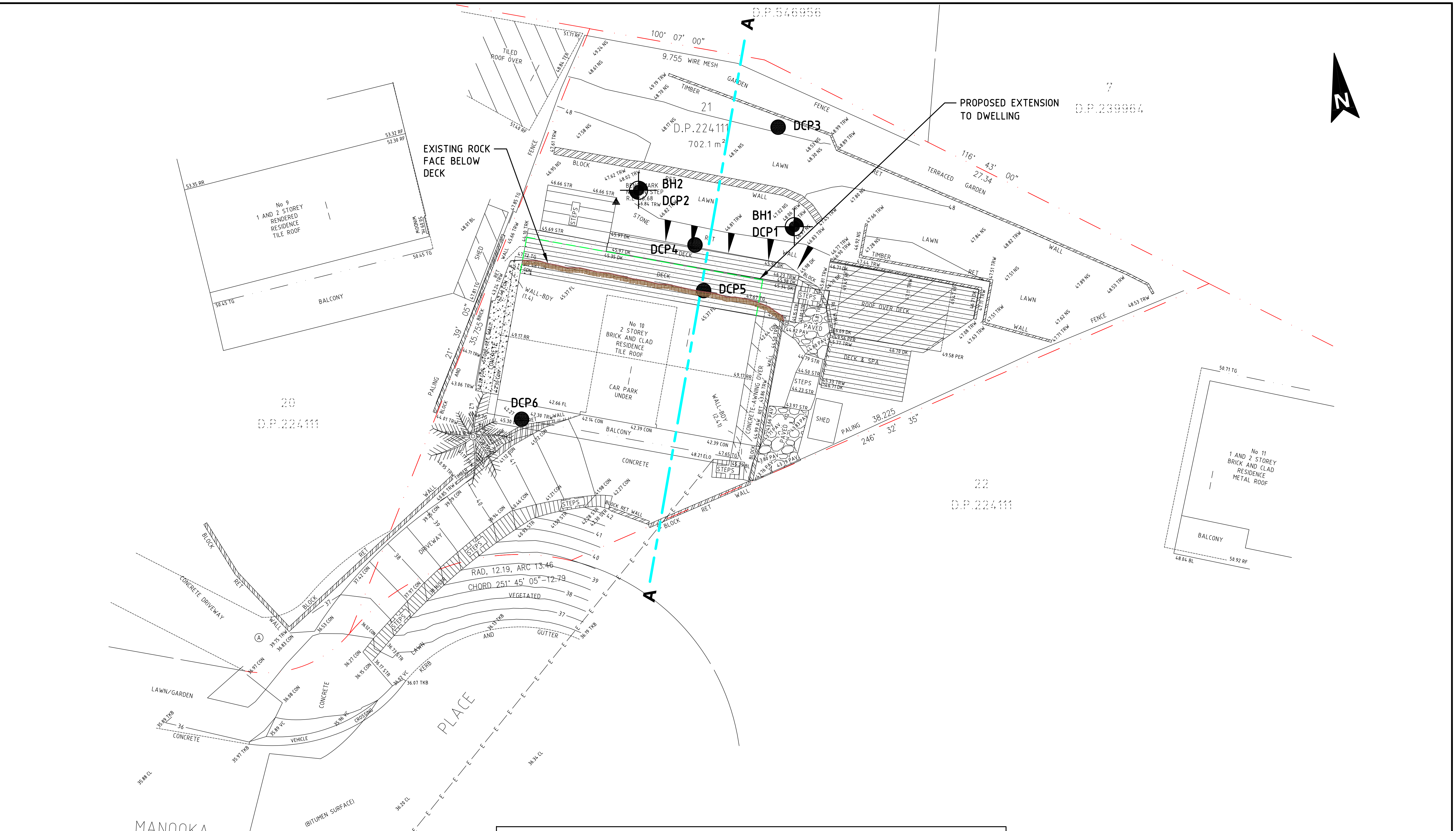
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).

Appendix 2



VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered	fg - Fine Grained
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered	mg - Medium Grained
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered	cg - Coarse Grained
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered	MAS - Massive
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered	BD - Bedded
	H - Hard	VHS - Very High Strength	FR - Fresh	OC - Outcrop

0 2 4 6 8 10m

4 3 2 1

SCALE 1:200

SITE PLAN AND TEST LOCATIONS FIGURE 1

CROZIER
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Crozier Geotechnical
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Brookvale NSW 2100
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Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

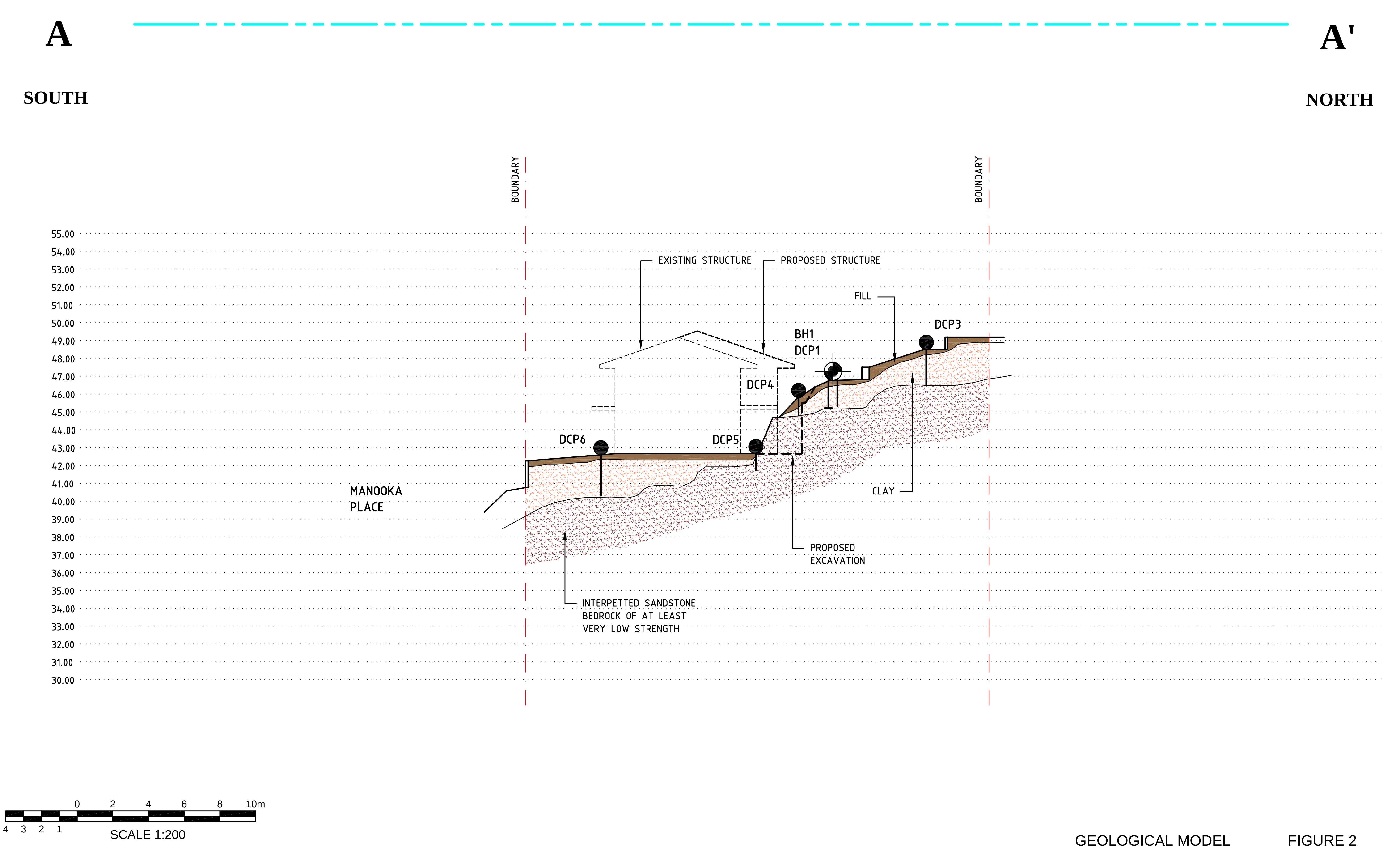
LEGEND

- BH- AUGER LOCATION
- BH- DCP- AUGER / DYNAMIC PENETRATION TEST
- DCP- DYNAMIC PENETRATION TEST
- A—A' CROSS SECTION REFERENCE LINE
- BOUNDARY LINE

SCALE :	1:200
DRAWING :	FIGURE 1
DATE :	10.07.21
APPROVED BY :	J.D.
DRAWN BY :	A.C.W.
PROJECT :	2021-144

PREPARED FOR :
TIMOTHY & DANIELLE O'CALLAGHAN

ADDRESS :
10 MANOOKA PLACE
WARRIEWOOD, N.S.W.



GEOLOGICAL MODEL

FIGURE 2

CROZIER
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Crozier Geotechnical
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Brookvale NSW 2100

ABN: 96 113 453 624
Phone: (02) 9939 1882
Email: info@croziergeotech.com.au

Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

LEGEND

- AUGER LOCATION
- AUGER / DYNAMIC PENETRATION TEST
- DCP- DYNAMIC PENETRATION TEST
- BOUNDARY LINE
- FILL
- CLAY
- SANDSTONE BEDROCK

SCALE :	1:200
DRAWING :	FIGURE 2
DATE :	10.07.21
APPROVED BY :	J.D.
DRAWN BY :	A.C.W.
PROJECT :	2021-144

PREPARED FOR :
TIMOTHY & DANIELLE O'CALLAGHAN

ADDRESS :
10 MANOOKA PLACE
WARRIEWOOD, N.S.W.

BOREHOLE LOG

CLIENT: Timothy & Danielle O'Callaghan **DATE:** 30/06/2021 **BORE No.:** 1

PROJECT: Additions and Alterations **PROJECT No.:** 2021-144 **SHEET:** 1 of 1

LOCATION: 10 Manooka Place Warriewood **SURFACE LEVEL:** RL47.00m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		TOPSOIL/FILL: loose, brown, fine to medium grained, moist, Silty/sandy topsoil with sandstone and ironstone gravels				
0.30	CI	CLAY: Stiff, orange brown, medium plasticity, moist silty clay with sandstone and ironstone gravels	D	0.40 0.50		
0.65		Auger refusal @ 0.65m on very stiff clay				
1.00						
2.00						

RIG: Not applicable DRILLER: AC LOGGED JD

METHOD: Hand Auger

GROUND WATER OBSERVATIONS: not encountered

REMARKS: CHECKED:

BOREHOLE LOG

CLIENT: Timothy & Danielle O'Callaghan | **DATE:** 30/06/2021 | **BORE No.:** 2

PROJECT: Additions and Alterations | **PROJECT No.:** 2021-144 | **SHEET:** 1 of 1

LOCATION: 10 Manooka Place Warriewood | **SURFACE LEVEL:** RL47.00m

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		TOPSOIL/FILL: loose, brown, fine to medium grained, moist, Silty/sandy topsoil with sandstone and ironstone gravels				
0.35	CI	CLAY: Stiff, orange brown, medium plasticity, moist silty clay with sandstone and ironstone gravels				
			D	0.55		
0.65		Auger refusal @ 0.65m on hard low plasticity clay		0.65		
1.00						
2.00						

RIG: Not applicable | DRILLER: AC

METHOD: Hand Auger | LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: | CHECKED:

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Timothy & Danielle O'Callaghan **DATE:** 30/06/2021

PROJECT: Additions and Alterations **PROJECT No.:** 2021-144

LOCATION: 10 Manooka Place **SHEET:** 1 of 1

	Test Location							
Depth (m)	1	2	3	4	5	6		
0.00 - 0.10	1	2	1	-	20	1		
0.10 - 0.20	4	1	1	1	B@0.1	1		
0.20 - 0.30	8	2	2	1		0		
0.30 - 0.40	4	3	2	1		1		
0.40 - 0.50	2	5	3	0		8		
0.50 - 0.60	3	9	4	1		5		
0.60 - 0.70	4	12	11	2		8		
0.70 - 0.80	9	18	10	2		3		
0.80 - 0.90	5	14	9	3		3		
0.90 - 1.00	5	12	9	3		8		
1.00 - 1.10	9	9	11	3		8		
1.10 - 1.20	15	12	11	4		3		
1.20 - 1.30	22	B@1.2	13	6		3		
1.30 - 1.40	25		15	11		20		
1.40 - 1.50	End		B@1.4	B@1.4		11B@1.45		
1.50 - 1.60								
1.60 - 1.70								
1.70 - 1.80								
1.80 - 1.90								
1.90 - 2.00								
2.00 - 2.10								
2.10 - 2.20								
2.20 - 2.30								
2.30 - 2.40								
2.40 - 2.50								
2.50 - 2.60								
2.60 - 2.70								
2.70 - 2.80								
2.80 - 2.90								
2.90 - 3.00								

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide <1m³) from soils at crest of excavation		Appears up to 1.50m of soil along side western boundary	a) Shed adjacent to boundary 1.0m from edge of excavation, impact shed only b) Proposed structure will be in excavation, 0.5m from excavation face of height 3.5m, negligible damage to external walls		a) Person in shed 0.25hrs/day avge. b) Person in house 10 hrs/ day avge.	a) Likely to not evacuate b) Possible to not evacuate	a) Person in shed, potentially buried b) Person in house, minor damage only	
			Likely	**Prob. of Impact**	**Impacted**				
		a) Shed/garden of No. 9 Manooka Place	0.01	0.20	0.50	0.0104	0.75	1.0	**7.81E-06**
		b) Site structure	0.01	1.00	0.01	0.4167	0.5	0.1	**2.08E-06**
B	Landslip (rockslide/topple >5m³) of bedrock around perimeter of excavation due to poorly oriented defects		Exposed bedrock within excavation up to 2.1m high	a) Shed adjacent to boundary 1.0m from edge of excavation, impact shed b) Proposed structure will be in excavation, 0.5m from excavation face of height 3.5m, potential structural damage c) House >4.00m from western edge of 3.50m deep excavation, minor damage only		a) Person in shed 0.25hrs/day avge. b) Person in house 10 hrs/ day avge. c) Person in house 10 hrs/day avge.	a) Likely to not evacuate b) Possible to not evacuate c) Possible not to evacuate	a) Person in shed, buried b) Person in house, damage to walls, potentially crushed c) Person in house, minor damage only	
			Unlikely	**Prob. of Impact**	**Impacted**				
		a) Shed/garden No. 9 Manooka Place	0.0001	0.20	1.00	0.0104	0.75	1.0	**1.56E-07**
		b) Existing site structure	0.0001	1.00	0.20	0.4167	0.5	1.0	**4.17E-06**
		c) House of No. 9 Manooka Place	0.0001	0.1	0.05	0.4167	0.5	0.1	**1.04E-08**
C	Landslip (rockslide/topple <1m³) of bedrock around perimeter of excavation due to poorly oriented defects		Exposed bedrock within excavation up to 2.1m high	a) Shed adjacent to boundary 1.0m from edge of excavation, impact shed only b) Proposed structure will be in excavation, 0.5m from excavation face of height 3.5m, minor damage to external walls		a) Person in shed 0.25hrs/day avge. b) Person in house 10 hrs/ day avge.	a) Likely to not evacuate b) Possible to not evacuate	a) Person in shed, potentially buried b) Person in house, minor damage only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Shed/garden of No. 9 Manooka Place	0.001	0.20	0.50	0.0104	0.75	1.0	**7.81E-07**
		b) Existing site structure	0.001	1.00	0.05	0.4167	0.5	0.1	**1.04E-06**

* hazards considered in current condition and/or without remedial/stabilisation measures or poor support systems
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).
Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of house structure such as 1 bedroom (5%), where as large boulder roll may damage/destroy >50%)
* neighbouring houses considered for impact of slide to bedroom unless specified, due to high occupancy and lower potential for evacuation.
* considered for person most at risk, where multiple people occupy area then increased risk levels
* for excavation induced landslip then considered for adjacent premises/buildings founded off shallow footings, unless indicated
* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide <1m³) from soils at crest of excavation	a) Shed/garden of No. 9 Manooka Place	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Insignificant
		b) Site structure	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low
B	Landslip (rockslide/topple >5m³) of bedrock around perimeter of excavation due to poorly oriented defects	a) Shed/garden No. 9 Manooka Place	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site or MINOR damage to neighbouring property, requires large stabilising works .	Low
		b) Existing site structure	Unlikely	The event might occur under very adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site or MINOR damage to neighbouring property, requires large stabilising works .	Moderate
		c) House of No. 9 Manooka Place	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site or MINOR damage to neighbouring property, requires large stabilising works .	Low
C	Landslip (rockslide/topple <1m³) of bedrock around perimeter of excavation due to poorly oriented defects	a) Shed/garden of No. 9 Manooka Place	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Insignificant
		b) Existing site structure	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low

* hazards considered in current condition, without remedial/stabilisation measures and during construction works.

* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.

* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.

* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.

* Cost of site development estimated at $5,000,000

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the open drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter. Owner to check and flush retaining wall drainage pipes/systems	Every year or following each major rainfall event. Every 7 years or where dampness/moisture
Retaining Walls. or remedial measures	Owner to inspect walls for deveation from as constructed condition and repair/replace. Replace non engineered rock/timber walls prior to collapse	Every two years or following major rainfall event. As soon as practicable
Large Trees on or adjacent to site	Arborist to check condition of trees and remove as required. Where tree within steep slopes (>18°) or adjacent to structures requires geotechincal inspection prior to removal	Every five years
Slope Stability	Geotechnical Engineering Consultant to check on site stability and maintenance	Five years after construction is completed.

N.B. Provided the above shedule is maintained the design life of the property should conform with Councils Risk Management Policy.

Appendix 4

APPENDIX A

DEFINITION OF TERMS

INTERNATIONAL UNION OF GEOLOGICAL SCIENCES WORKING GROUP ON LANDSLIDES, COMMITTEE ON RISK ASSESSMENT

Risk – A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.

Hazard – A condition with the potential for causing an undesirable consequence (*the landslide*). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.

Elements at Risk – Meaning the population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.

Probability – The likelihood of a specific outcome, measured by the ratio of specific outcomes to the total number of possible outcomes. Probability is expressed as a number between 0 and 1, with 0 indicating an impossible outcome, and 1 indicating that an outcome is certain.

Frequency – A measure of likelihood expressed as the number of occurrences of an event in a given time. See also Likelihood and Probability.

Likelihood – used as a qualitative description of probability or frequency.

Temporal Probability – The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.

Vulnerability – The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

Consequence – The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.

Risk Analysis – The use of available information to estimate the risk to individuals or populations, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification, and risk estimation.

Risk Estimation – The process used to produce a measure of the level of health, property, or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis, and their integration.

Risk Evaluation – The stage at which values and judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, and economic consequences, in order to identify a range of alternatives for managing the risks.

Risk Assessment – The process of risk analysis and risk evaluation.

Risk Control or Risk Treatment – The process of decision making for managing risk, and the implementation, or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.

Risk Management – The complete process of risk assessment and risk control (*or risk treatment*).

Individual Risk – The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.

Societal Risk – The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental, and other losses.

Acceptable Risk – A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.

Tolerable Risk – A risk that society is willing to live with so as to secure certain net benefits in the confidence that it is being properly controlled, kept under review and further reduced as and when possible.

In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk even though they recognise it is not properly controlled.

Landslide Intensity – A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, kinetic energy per unit area.

Note: Reference should also be made to Figure 1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	**Notional Boundary**					
10^{-1}	5×10^{-2}	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	5×10^{-3}	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	5×10^{-4}	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	5×10^{-5}	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	5×10^{-6}	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

Note: (1) The table should be used from left to right; use Approximate Annual Probability or Description to assign Descriptor, not *vice versa*.

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate Cost of Damage		Description	Descriptor	Level
Indicative Value	**Notional Boundary**			
200%	100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
60%	40%	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	10%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%		Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

Notes: (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.

(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.

(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa

APPENDIX C: – QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (CONTINUED)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)				
	Indicative Value of Approximate Annual Probability	**1: CATASTROPHIC 200%**	**2: MAJOR 60%**	**3: MEDIUM 20%**	**4: MINOR 5%**	**5: INSIGNIFICANT 0.5%**
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or **L** (5)
B - LIKELY	10^{-2}	VH	VH	H	M	**L**
C - POSSIBLE	10^{-3}	VH	H	M	M	**VL**
D - UNLIKELY	10^{-4}	H	M	**L**	**L**	**VL**
E - RARE	10^{-5}	M	**L**	**L**	**VL**	**VL**
F - BARELY CREDIBLE	10^{-6}	**L**	**VL**	**VL**	**VL**	**VL**

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

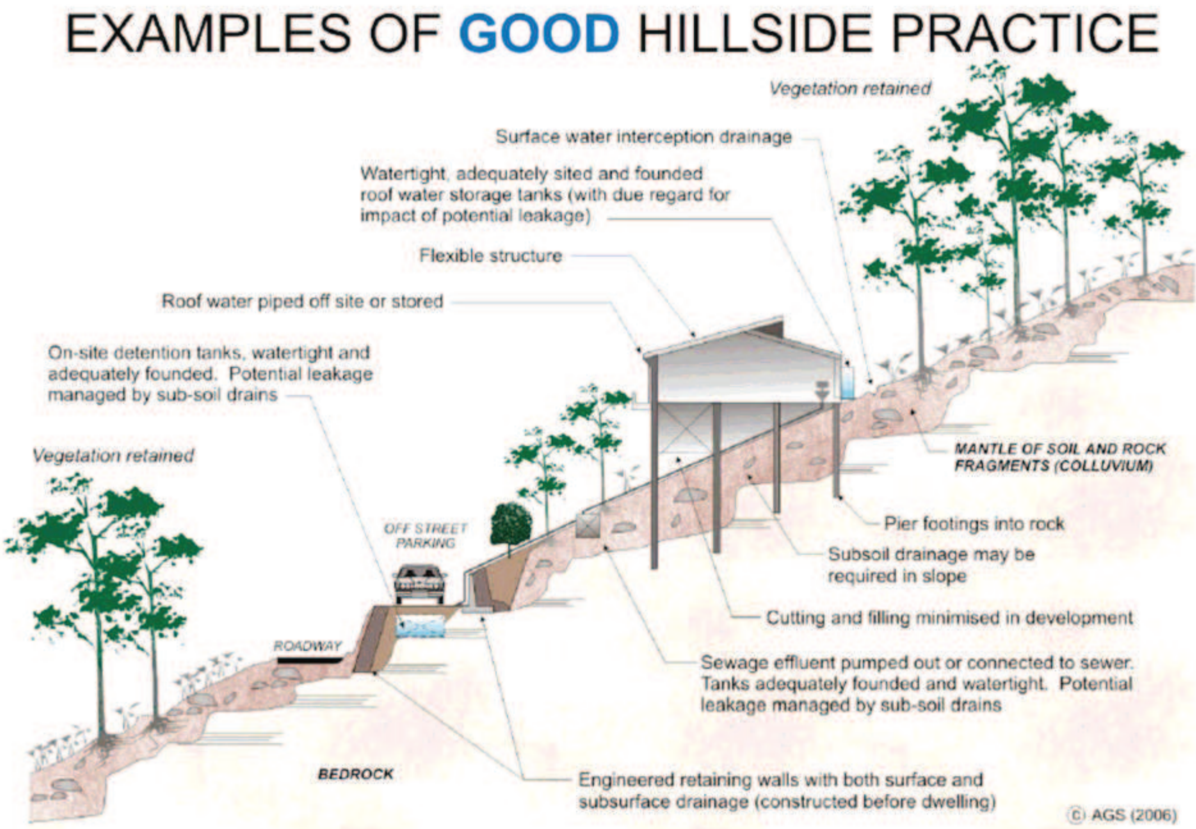
Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

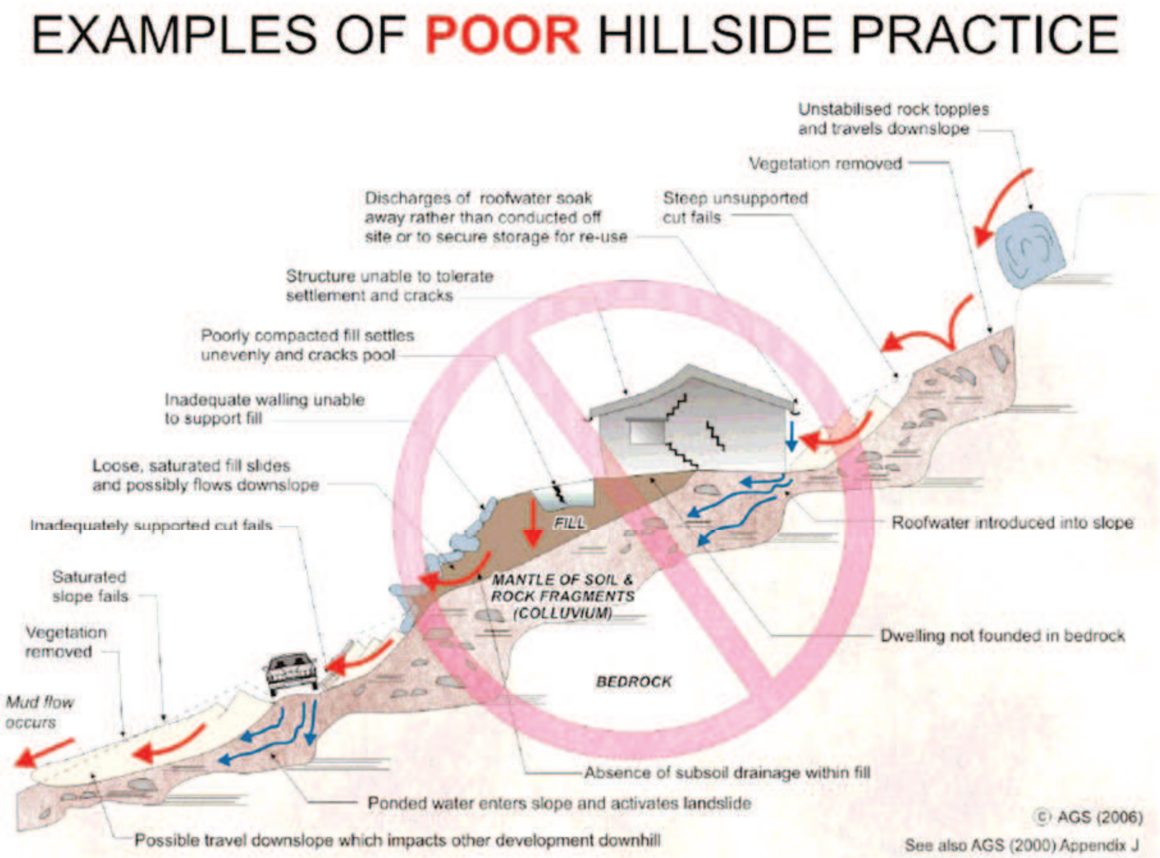
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Appendix 5

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	GOOD ENGINEERING PRACTICE	*POOR ENGINEERING PRACTICE*
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical practitioner at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminatory bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance including onto property below. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within rock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide general falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge roof runoff into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by geotechnical consultant	
SITE VISITS	Site Visits by consultant may be appropriate during construction/	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident see advice. If seepage observed, determine causes or seek advice on consequences.	


EXAMPLES OF GOOD HILLSIDE PRACTICE
Vegetation retained
Surface water interception drainage
Watertight, adequately sited and founded roof water storage tanks (with due regard for impact of potential leakage)
Flexible structure
Roof water piped off site or stored
On-site detention tanks, watertight and adequately founded. Potential leakage managed by sub-soil drains
Vegetation retained
OFF STREET PARKING
ROADWAY
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
Sewage effluent pumped out or connected to sewer. Tanks adequately founded and watertight. Potential leakage managed by sub-soil drains
BEDROCK
Engineered retaining walls with both surface and subsurface drainage (constructed before dwelling)
© AGS (2006)


EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles unevenly and cracks pool
Inadequate walling unable to support fill
Loose, saturated fill slides and possibly flows downslope
Inadequately supported cut fails
FILL
Roofwater introduced into slope
Saturated slope fails
MANTLE OF SOIL & ROCK FRAGMENTS (COLLUVIUM)
Vegetation removed
Dwelling not founded in bedrock
Mud flow occurs
BEDROCK
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
Possible travel downslope which impacts other development downhill
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See also AGS (2000) Appendix J